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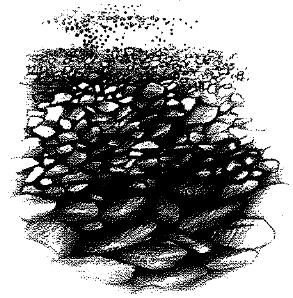

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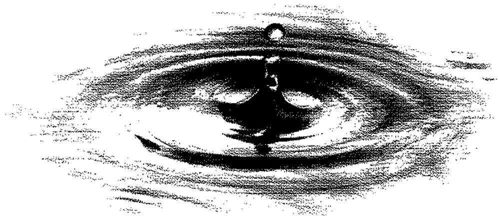

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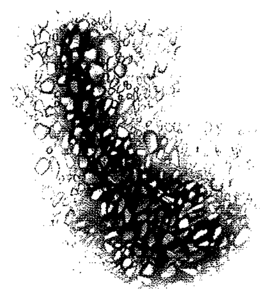
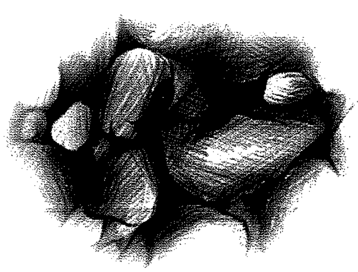


Environmental Assessments and Remediation

Sustainable Development Services



Geologic Assessments

Associated Earth Sciences, Inc.

Celebrating 25 Years of Service

Subsurface Exploration, Geologic Hazard, and Geotechnical Engineering Report

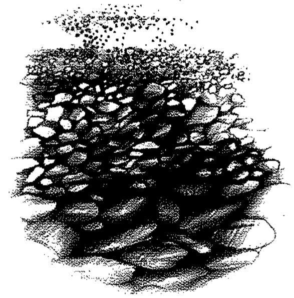
LITTLE PROPERTY

Shoreline, Washington

Prepared for

Preview Properties

Project No. KE05680A
February 24, 2006



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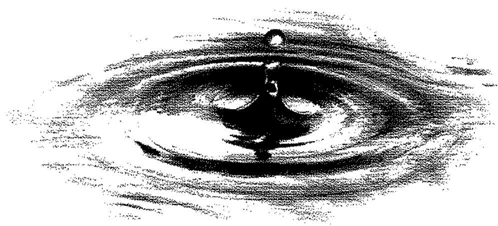

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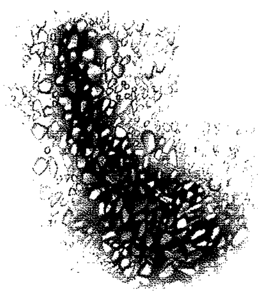
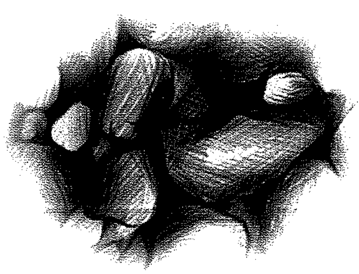


Environmental Assessments and Remediation

Sustainable Development Services



Geologic Assessments

Associated Earth Sciences, Inc.

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Subsurface Exploration, Geologic Hazard, and Geotechnical Engineering Report

LITTLE PROPERTY

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February 24, 2006

Associated Earth Sciences, Inc.

Celebrating 25 Years of Service

February 24, 2006
Project No. KE05680A

Preview Properties
1222 185th Street NE, Suite 102
Shoreline, Washington 98422

Attention: Mr. Bill Young

Subject: Subsurface Exploration, Geologic Hazard, and
Geotechnical Engineering Report
Little Property
14521 11th Avenue NE
Shoreline, Washington

Dear Mr. Young:

Associated Earth Sciences, Inc. (AESI) is pleased to present the enclosed copies of the above-referenced report. This report summarizes the results of our subsurface exploration, geologic hazard, and geotechnical engineering studies and offers preliminary recommendations for the design and development of the proposed project. Our recommendations are preliminary because building and grading plans for the project had not yet been prepared at the time of this report.

We have enjoyed working with you on this study and are confident that the recommendations presented in this report will aid in the successful completion of your project. If you should have any questions or if we can be of additional help to you, please do not hesitate to call.

Sincerely,
ASSOCIATED EARTH SCIENCES, INC.
Kirkland, Washington

Jon N. Sondergaard, P.G., P.E.G.
Principal Geologist

cc: D.R. Strong Consulting Engineers, Inc., 10604 NE 38th Place, Suite 101, Kirkland, WA 98033

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SUBSURFACE EXPLORATION, GEOLOGIC HAZARD, AND GEOTECHNICAL ENGINEERING REPORT

LITTLE PROPERTY

Shoreline, Washington

Prepared for:
Preview Properties
1222 185th Street NE, Suite 102
Shoreline, Washington 98422

Prepared by:
Associated Earth Sciences, Inc.
911 5th Avenue, Suite 100
Kirkland, Washington 98033
425-827-7701
Fax: 425-827-5424

February 24, 2006
Project No. KE05680A

I. PROJECT AND SITE CONDITIONS

1.0 INTRODUCTION

This report presents the results of our subsurface exploration, geologic hazard, and geotechnical engineering study for construction of a new single-family, residential subdivision located at 14521 11th Avenue NE in Shoreline, Washington (Figure 1). The proposed subdivision will consist of four to five building lots with associated roads and utilities. In the event that any changes in the nature, design, or location of the proposed project are planned, the conclusions and recommendations contained in this report should be reviewed and modified, or verified, as necessary.

1.1 Purpose and Scope

The purpose of this study was to provide subsurface data to be utilized in design and construction of the new subdivision at the above-referenced site. Our study included a review of available geologic literature, excavating five exploration pits, and performing geologic studies to assess the type, thickness, distribution, and physical properties of the subsurface sediments and shallow ground water conditions. Geotechnical engineering studies were also conducted to determine allowable foundation soil bearing pressures, suitable types of foundations, and recommendations for site preparation, geologic hazard mitigation, drainage considerations, and erosion control. This report summarizes our current fieldwork and offers preliminary geotechnical engineering recommendations based on our present understanding of the project. Our recommendations are preliminary because building and grading plans for the project had not yet been prepared at the time of this report.

1.2 Authorization

Written authorization to proceed with this study was granted by Mr. Bill Young of Preview Properties. Our study was based on our visit to the site and accomplished in general accordance with our scope of work letter dated September 21, 2005. This report has been prepared for the exclusive use of the Mr. Bill Young, Preview Properties, and their agents for specific application to this project. Within the limitations of scope, schedule, and budget, our services have been performed in accordance with generally accepted geotechnical engineering and engineering geology practices in effect in this area at the time our report was prepared. No other warranty, express or implied, is made. It must be understood that no recommendations or engineering design can yield a guarantee of stable slopes. Our observations, findings, and opinions are a means to identify and reduce the inherent risks to the owner.

2.0 PROJECT AND SITE DESCRIPTION

This report was completed with an understanding of the project based on our discussions with Mr. Young and a topographic survey of the property prepared by D.R. Strong Consulting Engineers, Inc. (D.R. Strong) dated July 13, 2005. Present plans call for the construction of four to five single-family, residential houses on the subject property.

The property is situated at 14521 11th Avenue NE in Shoreline, Washington. The approximately 69,325 square foot parcel consists of Lots 13 and 14 of Paramount Park, Division 2. The property is bordered by NE 145th Street to the south, unimproved 10th Avenue NE to the west, the 11th Avenue NE right-of-way to the east, and a single-family residence and undeveloped property to the north. The parcel occupies a topographic knob that slopes down to the west along the west and north sides, down to the east along the east side, and down to the south along the south side. The slopes on the north and west are natural steep slopes, the slope on the south is the road-cut created by construction of NE 145th Street, and the slopes on the east are gentler and somewhat modified by past site use. Total elevation change across the property was on the order of 52 feet. A house that used to occupy the center of Lot 13 has been removed, but several sheds remain on the property.

3.0 SUBSURFACE EXPLORATION

Our field study included excavating five exploration pits and performing a geologic hazard reconnaissance to gain information about the site. The approximate locations of the exploration pits are shown on the Site and Exploration Plan, Figure 2. The various types of sediments, as well as the depths where characteristics of the sediments changed, are indicated on the exploration logs presented in the Appendix. The depths indicated on the logs where conditions changed may represent gradational variations between sediment types. Our explorations were approximately located in the field by measuring from known site features shown on a topographic survey prepared by D.R. Strong dated July 13, 2005.

The conclusions and recommendations presented in this report are based on the five exploration pits, site reconnaissance, and review of applicable geologic literature completed for this study. The number, locations, and depths of the explorations were completed within site and budgetary constraints. Because of the nature of exploratory work below ground, extrapolation of subsurface conditions between field explorations is necessary. It should be noted that differing subsurface conditions might sometimes be present due to the random nature of deposition and the alteration of topography by past grading and/or filling. The nature and extent of any variations between the field explorations may not become fully evident until construction. If variations are observed at that time, it may be necessary to re-evaluate specific recommendations in this report and make appropriate changes.

3.1 Exploration Pits

Exploration pits were excavated with a tractor-mounted backhoe. The pits permitted direct, visual observation of subsurface conditions. Materials encountered in the exploration pits were studied and classified in the field by an engineering geologist from our firm. All exploration pits were backfilled immediately after examination and logging. Selected samples were then transported to our laboratory for further visual classification and testing, as necessary.

4.0 SUBSURFACE CONDITIONS

Subsurface conditions on the parcel were inferred from the field explorations accomplished for this study, visual reconnaissance of the site, and review of applicable geologic literature. As shown on the field logs, the exploration pits generally encountered natural deposits consisting of medium dense to very dense, silty gravelly sand. Fill was encountered in the vicinity of the former house location. The following section presents more detailed subsurface information organized from the shallowest (youngest) to the deepest (oldest) sediment types.

4.1 Stratigraphy

Fill

Fill soils (those not naturally placed) were encountered in exploration pits EP-4 and EP-5 completed near the former house location. The fill ranged in thickness from 3 to 4 feet in EP-5 and EP-4, respectively. As noted on the exploration logs, the fill typically consisted of loose to medium dense, moist to saturated, brown, silty, gravelly, fine to medium sand with scattered organics and rubble. These materials appear to vary in both quality and depth across the site. Since the quality, thickness, and compaction of the fill materials are low or variable, the fill is unsuitable for structural support.

Till

Natural soils beneath the fill materials, and at the surface where fill materials were absent, consisted of glacial till. The till sequence encountered within our site explorations typically consisted of medium dense to dense, rusty brown, silty gravelly sand to sandy silt with scattered cobbles (weathered lodgement till) extending to depths of approximately 2 to 3 feet below the existing ground surface. Underlying these soils, very dense, gray, silty gravelly sand (lodgement till) was encountered. This material was overrun by several thousand feet of ice during the last glacial advance that resulted in a compact soil possessing high strength, low compressibility, and low permeability characteristics.

4.2 Hydrology

Ground water seepage was not encountered in any of our exploration pits at the time of our field study in February 2006. Seepage may occur at random depths and locations in unsupervised or non-uniform fills. It should be noted that fluctuations in the level of the ground water may occur due to the time of the year, variations in the amount of precipitation, and changes in site development.

II. GEOLOGIC HAZARDS AND MITIGATIONS

The following discussion of potential geologic hazards is based on the geologic, slope, and ground water conditions as observed and discussed herein. The discussion will be limited to potential seismic, landslide, and erosion hazards. Chapter 20.80, Subchapter 2 of the *Shoreline Municipal Code* classifies Geologic Hazard Areas within the City of Shoreline. Based on this code, the subject site would be classified as a Landslide and Erosion Hazard Area.

5.0 SEISMIC HAZARDS AND RECOMMENDED MITIGATION

Earthquakes occur in the Puget Lowland with great regularity. Fortunately, the vast majority of these events are small and are usually not felt by people. However, large earthquakes do occur as evidenced by the 1949, 7.2-magnitude event; the 1965, 6.5-magnitude event; and the 2001, 6.8-magnitude event. The 1949 earthquake appears to have been the largest in this area during recorded history. Evaluation of earthquake return rates indicates that an earthquake of the magnitude between 5.5 and 6.0 likely will occur every 25 to 40 years in the Puget Sound area.

Generally, there are four types of potential geologic hazards associated with large seismic events: 1) surficial ground rupture, 2) seismically induced landslides, 3) liquefaction, and 4) ground motion. The potential for each of these hazards to adversely impact the proposed project is discussed below.

5.1 Surficial Ground Rupture

The nearest known fault trace to the project is the Seattle Fault. Recent studies by the U.S. Geological Survey (USGS) (e.g., Johnson et al., 1994, *Origin and Evolution of the Seattle Fault and Seattle Basin, Washington*, Geology, v. 22, p.71-74; and Johnson et al., 1999, *Active Tectonics of the Seattle Fault and Central Puget Sound Washington - Implications for Earthquake Hazards*, Geological Society of America Bulletin, July 1999, v. 111, n. 7, p. 1042-1053) suggest that a northern trace of an east-west trending thrust fault zone (Seattle Fault) may project about 5 miles south of the project site. The recognition of this fault is relatively new, and data pertaining to it are limited with the studies still ongoing. According to the USGS studies, the latest movement of this fault was about 1,100 years ago when about 20 feet of surficial displacement took place. This displacement can presently be seen in the form of raised, wave-cut beach terraces along Alki Point in West Seattle and Restoration Point at the south end of Bainbridge Island. The recurrence interval of movement along this fault system is still unknown, although it is hypothesized to be in excess of several thousand years. Due to the suspected long recurrence interval and distance from the subject property, the potential for surficial ground rupture is considered to be low during the expected life of the structures.

5.2 Seismically Induced Landslides

Due to the high strength of the glacial till soils and lack of ground water seepage on slopes surrounding the site, in our opinion, the potential for seismically induced landslides on the property is relatively low. Mitigations for potential seismically induced landslides are the same as those presented in Section 6.0 below for landslide hazards.

5.3 Liquefaction

The encountered stratigraphy has a low potential for liquefaction due to their dense state and absence of adverse ground water conditions.

5.4 Ground Motion

Based on the site stratigraphy and visual reconnaissance of the site, it is our opinion that earthquake damage to the proposed structures when founded on a suitable bearing stratum would likely be caused by the intensity and acceleration associated with the event. Structural design of buildings should follow 2003 *International Building Code* (IBC) standards using Site Class "C" as defined in Table 1615.1.1. The 2003 IBC seismic design parameters for short period (S_S) and 1-second period (S_1) spectral acceleration values were determined by the latitude and longitude of the project site using the USGS National Seismic Hazard Mapping Project website (http://earthquake.usgs.gov/hazmaps/). Based on the more current 2002 data, the USGS website interpolated ground motions at the project site to be 1.21g and 0.42g for building periods of 0.2 and 1.0 seconds, respectively, with a 2 percent chance of exceedance in 50 years.

6.0 LANDSLIDE HAZARDS AND MITIGATION

; NOW (F)

Based on Chapter 20.80.220 of the *Shoreline Municipal Code*, the majority of the slope on the west and northwest sides of the subject property would be classified as a Very High Hazard Area because the slope inclination is generally greater than 40 percent. According to Chapter 20.80.030(G), the slope on the south side of the property that was created by the construction of NE 145^{th} Street would be exempt from the critical area regulations because it was created through a prior, legal grading activity. Because the slope on the west and northwest is classified as a Very High Hazard Area, no alteration of this slope would be allowed (Chapter 20.80.240[B]).

To mitigate the risk of landslides impacting the proposed construction and adjacent properties, we recommend a minimum top of slope buffer of 15 feet be incorporated into the project design. In our opinion, reduction of the buffer from 50 feet to 15 feet, as allowed under Chapter 20.80.230(c) of the *Shoreline Municipal Code* is appropriate based on the high strength of the site soils (glacial till), the lack of ground water seepage on the slope, and the lack of previous, historic slide activity on the slope. As with all slopes, surface drainage should be properly controlled and directed away from sloping areas. Downspouts from roofs should be tightlined into suitable storm water drainage systems. At no time should fill be pushed over the top of bank. Uncontrolled fill over tops of slopes may promote landslides or debris flow activity.

There is No Reason to Not approve

No change in Water
No change in "over burden"

7.0 EROSION HAZARDS AND MITIGATION

To mitigate and reduce the erosion hazard potential and off-site sediment transport, a temporary erosion and sediment control plan should be prepared for the project, and we recommend the following:

1. Surface water should not be allowed to flow across the site over unprotected surfaces.

2. All storm water from impermeable surfaces, including driveways and roofs, should be tightlined to a suitable temporary storm water collection system.

3. Silt fences should be placed and maintained around the downslope perimeter of the proposed construction area throughout the entire construction phase of the project until permanent landscaping and permanent storm water collection facilities have been installed.

4. Soils that are to be reused around the site should be stored in such a manner as to reduce erosion from the stockpile. Protective measures may include, but are not necessarily limited to, covering with plastic sheeting, the use of low stockpiles in flat areas, or the use of straw bales and/or additional silt fences around pile perimeters. Soils should not be stockpiled on the steeply sloping portions of the property.

5. Areas stripped of natural vegetation during construction should be replanted as soon as possible or otherwise protected.

III. PRELIMINARY DESIGN RECOMMENDATIONS

8.0 INTRODUCTION

Our exploration indicates that, from a geotechnical standpoint, the parcel is suitable for the proposed development provided the risks discussed are accepted and the preliminary recommendations contained herein are properly followed. The bearing stratum is relatively shallow, and spread footing foundations may be utilized. We understand that the distribution of foundation loads of the wood-frame buildings will be typical; no concentrated loads are anticipated. Consequently, foundations bearing upon structural fill or the natural, dense, glacial till are capable of providing suitable building support.

9.0 SITE PREPARATION

Old foundations presently on the site that are under building areas or not part of future plans should be removed. Any buried utilities should be removed or relocated if they are under building areas. The resulting depressions should be backfilled with structural fill, as discussed under the *Structural Fill* section.

Site preparation of planned building and road/parking areas should include removal of all trees, brush, debris, and any other deleterious material. Additionally, the upper organic topsoil should be removed and the remaining roots grubbed. Areas where loose surficial soils exist due to grubbing operations should be considered as fill to the depth of disturbance and treated as subsequently recommended for structural fill placement.

Loose surficial soils or old fill should be stripped down to the underlying, medium dense to very dense natural soil. Since the density of the soil is variable, random soft pockets may exist, and the depth and extent of stripping can best be determined in the field by the geotechnical engineer or his representative. This depth generally occurs at approximately 3 to 6 inches for topsoil and 3 to 4 feet for fill in the area of the former house. We recommend that road and drive areas be proof-rolled with a loaded dump truck to identify any soft spots; soft areas should be overexcavated and backfilled with structural fill.

In our opinion, stable construction slopes should be the responsibility of the contractor and should be determined during construction. For estimating purposes, we anticipate that temporary, unsupported cut slopes in the unsaturated natural soils can be made at a maximum slope of 1H:1V (Horizontal:Vertical). As is typical with earthwork operations, some sloughing and raveling may occur, and cut slopes may have to be adjusted in the field. In addition, WISHA/OSHA regulations should be followed at all times.

The on-site soils contain a high percentage of fine-grained material that makes them moisture-sensitive and subject to disturbance when wet. The Contractor must use care during site preparation and excavation operations so that the underlying soils are not softened. If disturbance occurs, the softened soils should be removed and the area brought to grade with structural fill. Consideration should be given to protecting access and staging areas with an appropriate section of crushed rock or asphalt treated base (ATB).

If crushed rock is considered for the access and staging areas, it should be underlain by engineering stabilization fabric to reduce the potential of fine-grained materials pumping up through the rock and turning the area to mud. The fabric will also aid in supporting construction equipment, thus reducing the amount of crushed rock required. We recommend that at least 10 inches of rock be placed over the fabric; however, due to the variable nature of the near-surface soils and differences in wheel loads, this thickness may have to be adjusted by the contractor in the field.

10.0 STRUCTURAL FILL

There is a possibility that structural fill will be necessary to establish desired grades. All references to structural fill in this report refer to subgrade preparation, fill type, and placement and compaction of materials as discussed in this section. If a percentage of compaction is specified under another section of this report, the value given in that section should be used.

If fill is to be placed on slopes steeper than 5H:1V, the base of the fill should be tied to firm, stable subsoil by appropriate keying and benching, which would be established in the field to suit the particular soil conditions at the time of grading. The keyway will act as a shear key to embed the toe of the new fill into the hillside. Generally, the keyway for hillside fills should be at least 8 feet wide and cut into the lower, dense sand or stiff silt. Level benches would then be cut horizontally across the hill following the contours of the slope. No specific width is required for the benches, although they are usually a few feet wider than the dozer being used to cut them. All fills proposed over a slope should be reviewed by our office prior to construction.

After overexcavation/stripping has been performed to the satisfaction of the geotechnical engineer or his representative, the upper 12 inches of exposed ground should be recompacted to a firm and unyielding condition, as determined by the geotechnical engineer or his representative. If the subgrade contains too much moisture, adequate recompaction may be difficult or impossible to obtain and should probably not be attempted. In lieu of recompaction, the area to receive fill should be blanketed with washed rock or quarry spalls to act as a capillary break between the new fill and the wet subgrade. Where the exposed ground remains soft and further overexcavation is impractical, placement of an engineering stabilization fabric may be necessary to prevent contamination of the free-draining layer by silt migration from below.

After the recompacted, exposed ground is tested and approved, or a free-draining rock course is laid, structural fill may be placed to attain desired grades. Structural fill is defined as non-organic soil, acceptable to the geotechnical engineer, placed in maximum 8-inch loose lifts with each lift being compacted to at least 95 percent of the modified Proctor maximum density using American Society for Testing and Materials (ASTM):D 1557 as the standard. In the case of roadway and utility trench filling, the backfill should be placed and compacted in accordance with current local or county codes and standards. The top of the compacted fill should extend horizontally outward a minimum distance of 3 feet beyond the location of the perimeter footings or roadway edges before sloping down at an angle of 2H:1V.

The contractor should note that any proposed fill soils must be evaluated by Associated Earth Sciences, Inc. (AESI) prior to their use in fills. This would require that we have a sample of the material 72 hours in advance to perform a Proctor test and determine its field compaction standard. Soils in which the amount of fine-grained material (smaller than the No. 200 sieve) is greater than approximately 5 percent (measured on the minus No. 4 sieve size) should be considered moisture-sensitive. Use of moisture-sensitive soil in structural fills should be limited to favorable dry weather conditions. The on-site soils generally contained significant amounts of silt and are considered moisture-sensitive. In addition, construction equipment traversing the site when the soils are wet can cause considerable disturbance. If fill is placed during wet weather or if proper compaction cannot be obtained, a select import material consisting of a clean, free-draining gravel and/or sand should be used. Free-draining fill consists of non-organic soil with the amount of fine-grained material limited to 5 percent by weight when measured on the minus No. 4 sieve fraction.

A representative from our firm should inspect the stripped subgrade and be present during placement of structural fill to observe the work and perform a representative number of in-place density tests. In this way, the adequacy of the earthwork may be evaluated as filling progresses and any problem areas may be corrected at that time. It is important to understand that taking random compaction tests on a part-time basis will not assure uniformity or acceptable performance of a fill. As such, we are available to aid the owner in developing a suitable monitoring and testing frequency.

11.0 FOUNDATIONS

Spread footings may be used for building support when founded on medium dense to dense natural soils (weathered till and till) or structural fill placed as previously discussed. We recommend that an allowable bearing pressure of 2,500 pounds per square foot (psf) be utilized for design purposes, including both dead and live loads. An increase of one-third may be used for short-term wind or seismic loading. Perimeter footings should be buried at least 18 inches into the surrounding soil for frost protection. However, all footings must penetrate to the prescribed bearing stratum, and no footing should be founded in or above loose, organic, or existing fill soils.

It should be noted that the area bounded by lines extending downward at 1H:1V from any footing must not intersect another footing or intersect a filled area that has not been compacted to at least 95 percent of ASTM:D 1557. In addition, a 1.5H:1V line extending down from any footing must not daylight because sloughing or raveling may eventually undermine the footing. Thus, footings should not be placed near the edge of steps or cuts in the bearing soils.

Anticipated settlement of footings founded on medium dense to dense natural soil or approved structural fill should be on the order of ¾ inch. However, disturbed soil not removed from footing excavations prior to footing placement could result in increased settlements. All footing areas should be observed by AESI prior to placing concrete to verify that the design bearing capacity of the soils has been attained and that construction conforms to the recommendations contained in this report. Such observations may be required by the governing municipality. Perimeter footing drains should be provided, as discussed under the section on *Drainage Considerations*.

12.0 LATERAL WALL PRESSURES

All backfill behind walls or around foundation units should be placed as per our recommendations for structural fill and as described in this section of the report. Horizontally backfilled walls that are free to yield laterally at least 0.1 percent of their height may be designed using an equivalent fluid equal to 35 pounds per cubic foot (pcf). Fully restrained, horizontally backfilled, rigid walls that cannot yield should be designed for an equivalent fluid of 50 pcf. If parking areas are adjacent to walls, a surcharge equivalent to 2 feet of soil should be added to the wall height in determining lateral design forces.

The lateral pressures presented above are based on the conditions of a uniform backfill consisting of on-site, silty gravelly sand compacted to 90 percent of ASTM:D 1557. A higher degree of compaction is not recommended, as this will increase the pressure acting on the wall. A lower compaction may result in settlement of structural features above the walls. Thus, the compaction level is critical and must be tested by our firm during placement. Surcharges from adjacent footings, heavy construction equipment, or sloping ground must be added to the above values. Perimeter footing drains should be provided for all retaining walls, as discussed under the section on *Drainage Considerations*.

It is imperative that proper drainage be provided so that hydrostatic pressures do not develop against the walls. This would involve installation of a minimum, 1-foot-wide blanket drain for the full wall height to within 1 foot of finished grade using imported, washed gravel against the walls.

12.1 Passive Resistance and Friction Factors

Retaining wall footings/keyways cast directly against undisturbed, dense soils in a trench may be designed for passive resistance against lateral translation using an equivalent fluid equal to 300 pcf. The passive equivalent fluid pressure diagram begins at the top of the footing; however, total lateral resistance should be summed only over the depth of the actual key (truncated triangular diagram). This value applies only to footings/keyways where concrete is placed directly against the trench sidewalls without the use of forms. If footings are placed on grade and then backfilled, the top of the compacted backfill must be horizontal and extend outward from the footing for a minimum lateral distance equal to three times the height of the backfill before tapering down to grade. With backfill placed as discussed, footings may be designed for passive resistance against lateral translation using an equivalent fluid equal to 300 pcf and the truncated pressure diagram discussed above. Passive resistance values include a factor of safety equal to 3 in order to reduce the amount of movement necessary to generate passive resistance.

The friction coefficient for footings cast directly on undisturbed, dense soils may be taken as 0.35. This is an allowable value and includes a safety factor. Since it will be difficult to excavate these soils without disturbance, the soil under the footings must be recompacted to at least 95 percent of the above-mentioned standard for this value to apply.

13.0 FLOOR SUPPORT

A slab-on-grade floor may be used over structural fill or pre-rolled, medium dense natural ground. The floor should be cast atop a minimum of 4 inches of pea gravel, washed crushed rock, or other suitable material approved by the geotechnical engineer to act as a capillary break. It should also be protected from dampness by an impervious moisture barrier or otherwise sealed.

Another alternative would be to utilize a structural floor or crawl space-type construction. With this approach, floor support problems resulting from site disturbance are eliminated. If surficial soils are disturbed, the foundations can be excavated through the loose soils to suitable bearing and floor support is unaffected. Thus, a structural or crawl space floor is better-suited to wet weather construction than is slab-on-grade, although either system can be specified. In the case of a crawl space, the soil below the floor system should be covered with an impervious moisture barrier to reduce dampness.

14.0 DRAINAGE CONSIDERATIONS

The underlying, glacially compacted soils are relatively impermeable, and water will tend to perch atop this stratum. Additionally, traffic across these soils when they are damp or wet will result in disturbance of the otherwise firm stratum. Therefore, prior to site work and construction, the contractor should be prepared to provide subgrade protection and drainage, as necessary.

All retaining and footing walls should be provided with a drain at the footing elevation. Drains should consist of rigid, perforated, polyvinyl chloride (PVC) pipe surrounded by washed pea gravel. The level of the perforations in the pipe should be set down below the bottom of the footing at all locations, and the drains should be constructed with sufficient gradient to allow gravity discharge away from the buildings. In addition, all retaining walls should be lined with a minimum, 1-foot thick, washed gravel blanket provided over the full height of the wall to within 1 foot of finished grade, and which ties into the footing drain. If a drainage mat is used, it should include a minimum of 1 foot of free-draining, granular soil between the drainage mat and common wall backfill. Roof and surface runoff should not discharge into the footing drain system, but should be handled by a separate, rigid, tightline drain. In planning, exterior grades adjacent to walls should be sloped downward away from the structures to achieve surface drainage.

15.0 PROJECT DESIGN AND CONSTRUCTION MONITORING

At the time of this report, site grading, structural plans, and construction methods have not been finalized, and the recommendations presented herein are preliminary. We are available to provide additional geotechnical consultation as the project design develops and possibly changes from that upon which this report is based. We recommend that AESI perform a geotechnical review of the plans prior to final design completion. In this way, our earthwork and foundation recommendations may be properly interpreted and implemented in the design. This review is not included in our current scope of work and budget.

We are also available to provide geotechnical engineering and monitoring services during construction. The integrity of the foundations depends on proper site preparation and construction procedures. In addition, engineering decisions may have to be made in the field in the event that variations in subsurface conditions become apparent. Construction monitoring services are not part of this current scope of work. If these services are desired, please let us know and we will prepare a cost proposal.

We have enjoyed working with you on this study and are confident that these recommendations will aid in the successful completion of your project. If you should have any questions or require further assistance, please do not hesitate to call.

Sincerely,
ASSOCIATED EARTH SCIENCES, INC.
Kirkland, Washington

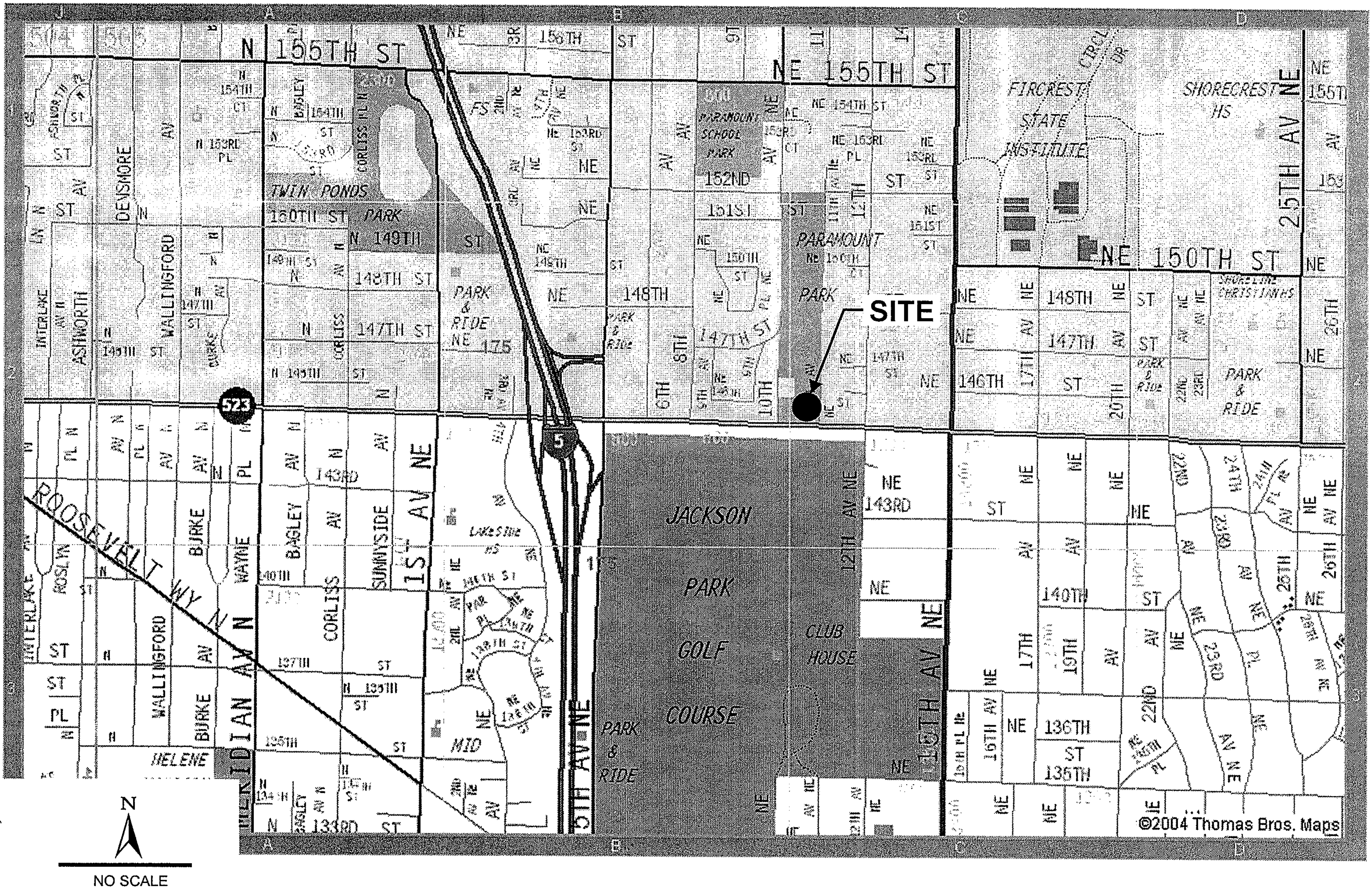
2/24/06

2-24-06

Jon N. Sondergaard, P.G., P.E.G.
Principal Engineering Geologist

Matthew A. Miller, P.E.
Associate Engineer

Attachments:	Figure 1:	Vicinity Map
	Figure 2:	Site and Exploration Plan
	Appendix:	Exploration Logs



Associated Earth Sciences, Inc.

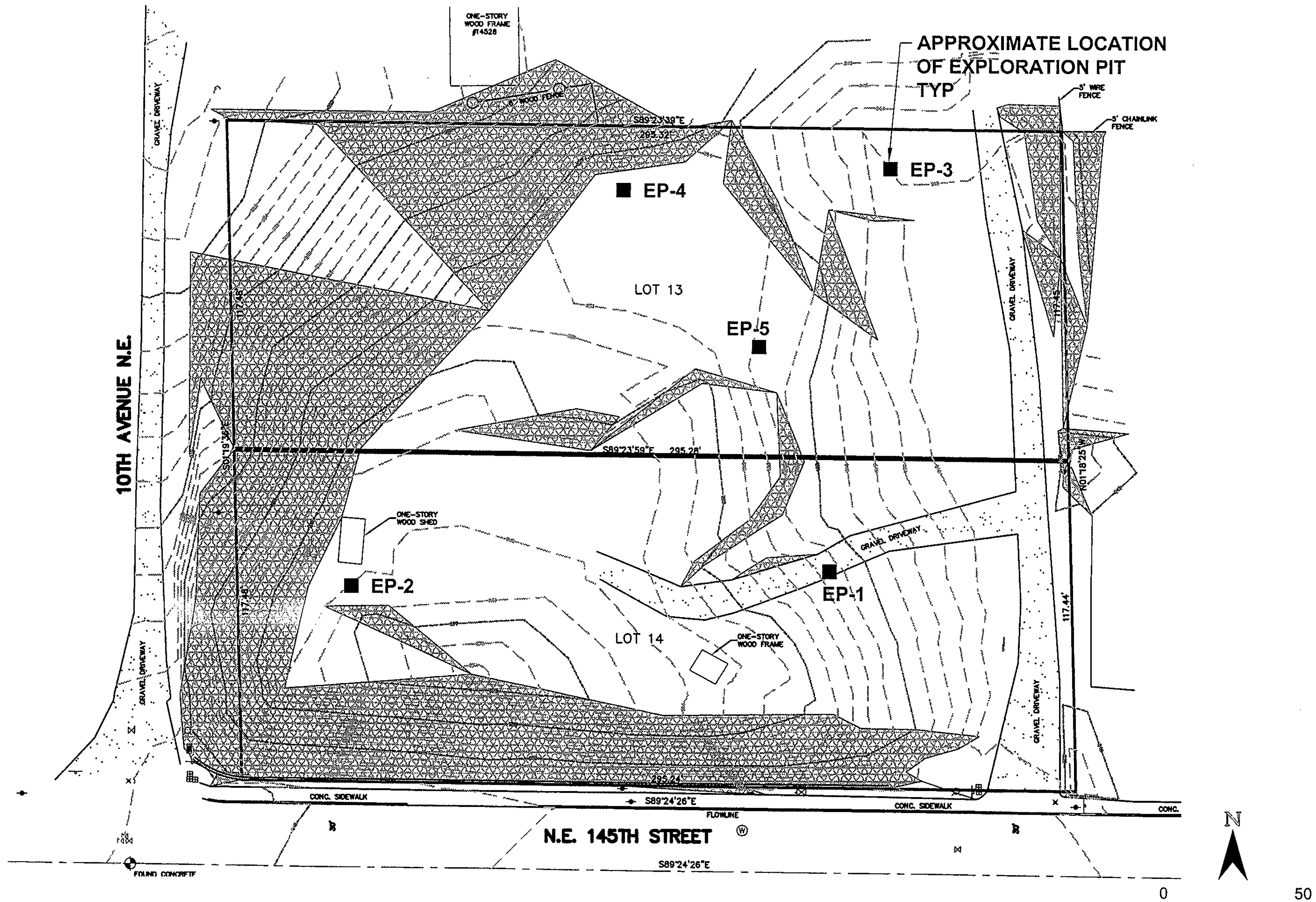
VICINITY MAP
NE 145th at 11th
SHORELINE, WASHINGTON

FIGURE 1

DATE 2/06

PROJ. NO. KE05680A

05680 NE 145th at 11th \ 05680 Vicinity.cdr



Reference: D.R. Strong, Consulting Engineers, Inc.

Associated Earth Sciences, Inc.

SITE AND EXPLORATION PLAN

NE 145TH STREET & 11TH
SHORELINE, WASHINGTON

FIGURE 2

DATE 2/06

PROJECT NO. KE05680A

05680 NE 141st Street and 11th \ 05680 Site and Explr.dwg

APPENDIX

LOG OF EXPLORATION PIT NO. EP-1

Depth (ft)	This log is part of the report prepared by Associated Earth Sciences, Inc. (AESI) for the named project and should be read together with that report for complete interpretation. This summary applies only to the location of this trench at the time of excavation. Subsurface conditions may change at this location with the passage of time. The data presented are a simplfication of actual conditions encountered. **DESCRIPTION**
	Topsoil/Duff
1	**Weathered Till** Medium dense to dense, moist, brown, slightly oxidized, silty gravelly SAND.
2	
3	
4	**Lodgement Till** Dense to very dense, moist, gray, silty gravelly SAND with scattered cobbles/boulders.
5	
6	
7	
8	Bottom of exploration pit at depth 7.5 feet No caving. No seepage.
9	
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KCTP3 05680A.GPJ February 7, 2006

145th Avenue NE & 11th
Shoreline, WA

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Project No. KE05680A

2/7/06

LOG OF EXPLORATION PIT NO. EP-2

Depth (ft)	This log is part of the report prepared by Associated Earth Sciences, Inc. (AESI) for the named project and should be read together with that report for complete interpretation. This summary applies only to the location of this trench at the time of excavation. Subsurface conditions may change at this location with the passage of time. The data presented are a simplification of actual conditions encountered. **DESCRIPTION**
	Lodgement Till
1	Dense to very dense, moist, gray, silty gravelly SAND with scattered cobbles and boulders.
2	
3	
4	
5	
6	
7	
8	
9	
10	
11	
12	Bottom of exploration pit at depth 11 feet Located at base of 6' cut for road. No caving. No seepage.
13	
14	
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LOG OF EXPLORATION PIT NO. EP-3

Depth (ft)	This log is part of the report prepared by Associated Earth Sciences, Inc. (AESI) for the named project and should be read together with that report for complete interpretation. This summary applies only to the location of this trench at the time of excavation. Subsurface conditions may change at this location with the passage of time. The data presented are a simplfication of actual conditions encountered. **DESCRIPTION**
1	**Weathered Till** Medium dense, moist, brown, silty SAND with scattered gravel.
2	**Lodgement Till**
3	Dense to very dense, moist, gray, silty SAND with gravel, with lenses of dense, moist, gray, gravelly SAND.
4	
5	
6	
7	
8	
9	
10	Bottom of exploration pit at depth 9 feet No caving. No seepage.
11	
12	
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LOG OF EXPLORATION PIT NO. EP-4

Depth (ft)	This log is part of the report prepared by Associated Earth Sciences, Inc. (AESI) for the named project and should be read together with that report for complete interpretation. This summary applies only to the location of this trench at the time of excavation. Subsurface conditions may change at this location with the passage of time. The data presented are a simplfication of actual conditions encountered. **DESCRIPTION**
	Fill
1	Loose, moist, silty SAND with scattered organics, concrete, and brick.
2	
3	
4	Buried topsoil at 4'.
	Weathered Till
5	Medium dense, moist, brown, silty gravelly SAND.
6	
7	
	Lodgement Till
8	Dense to very dense, moist, gray, silty gravelly SAND.
9	
10	Bottom of exploration pit at depth 9.5 feet No caving. No seepage.
11	
12	
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2/7/06

LOG OF EXPLORATION PIT NO. EP-5

Depth (ft)	This log is part of the report prepared by Associated Earth Sciences, Inc. (AESI) for the named project and should be read together with that report for complete interpretation. This summary applies only to the location of this trench at the time of excavation. Subsurface conditions may change at this location with the passage of time. The data presented are a simplfication of actual conditions encountered. **DESCRIPTION**
	Fill
1	Medium dense, moist, gray, silty gravelly SAND.
2	
3	2" relic topsoil at 3'.
	Weathered Till
4	Medium dense, moist, oxidized brown, silty SAND with gravel and scattered cobbles/boulders.
5	
6	
	Lodgement Till
7	Dense to very dense, moist, gray, silty gravelly SAND with scattered cobbles/boulders.
8	
9	
10	Bottom of exploration pit at depth 9 feet No caving. No seepage.
11	
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